



US Army Corps
of Engineers®

Weir Jetties at Coastal Inlets: Part 2, Case Studies

by William C. Seabergh and Leonette J. Thomas

PURPOSE: The Coastal and Hydraulics Engineering Technical Note (CHETN) herein provides information on the performance of selected weir jetty systems constructed in the United States and discussion of their functioning. A companion CHETN (Seabergh 2002) presents the design elements of weir jetty systems.

BACKGROUND: A weir jetty system is one of several methods for bypassing sediment at coastal inlets. The weir section, typically less than 1,000 ft long, is a depressed region of the jetty that permits waves, and the longshore current generated by wind, waves, and tide to transport sediments moving along the coast to enter a deposition basin located in the lee of the weir, thereby reducing the amount of sediment entering the navigation channel. A weir also acts as a breakwater and provides a semiprotected area for dredging the deposition basin. Another benefit is to allow flood currents to enter the inlet over the weir and through the channel during flood flow with subsequent channeling

of ebb flows out the navigation channel between the jetties. The flood currents are weaker in the navigation channel, relative to the channel ebb currents, promoting net seaward sediment flushing. Thus, less sediment enters the bay channels, where it is lost to the beach system if it settles in flood shoals in the bay or contributes additional volume in bay channels that require dredging. A potential benefit for new jetty systems is that the outer tips of the jetties may not need to extend seaward as far as a system without a weir jetty because seaward transport along the jetty is minimized (Seabergh and Lane 1977).

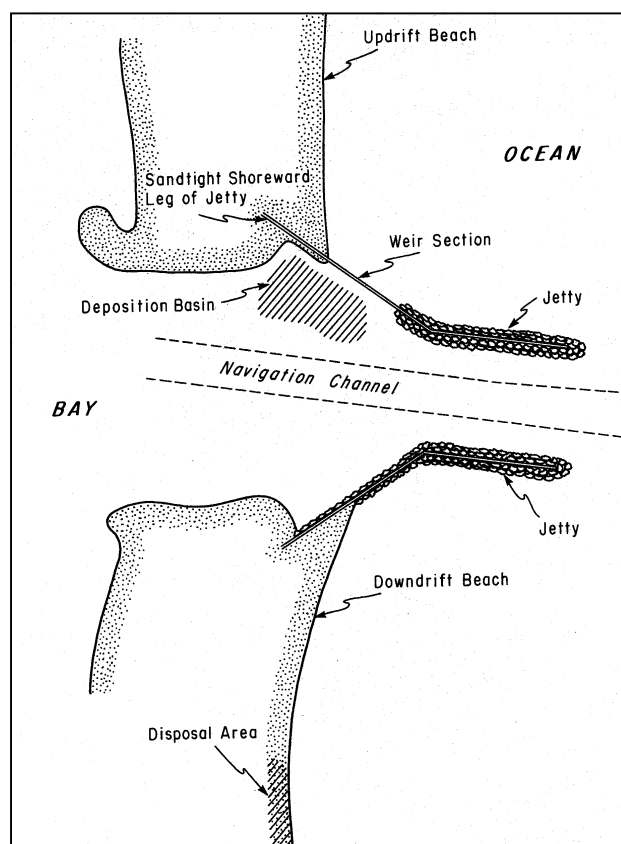


Figure 1. Typical elements of a weir jetty system

Figure 1 shows typical elements of a weir jetty system. The key elements of a weir jetty system are: (a) the navigation channel, (b) the jetty structures, (c) the weir structure, (d) the deposition basin, (e) the updrift beach, and (f) the downdrift beach. Many variations of this example (Figure 1) are possible dependent on structures orientation, bathymetry, and existence of bottom features such as shoals and rock reefs.

WEIR JETTY SYSTEMS: This CHETN reviews the performance of selected Federal and non-Federal weir jetty systems in the United States. Dimensions of key weir system elements and other information are presented in a summary table at the end of discussions of each project.

U.S. Army Corps of Engineers (USACE) documents, in particular Weggel (1981), provide detailed design guidance. Another useful reference with observations from a general physical model study of weir jetties is Seabergh (1983). Another summarizing reference is Weggel (1983).

WEIR JETTY SYSTEM LOCATED IN NORTH ATLANTIC DIVISION:

Rudee Inlet, VA. Because of the expanding popularity of Virginia Beach, VA, as a resort area in the mid-1950s, increased harbor facilities became necessary to satisfy boating interests. The accumulation of sand in the inlet caused navigational problems for charter fishing craft. This began an engineering approach to the development of Rudee Inlet by the city and Commonwealth of Virginia Beach. Beginning 1 July 1967 and ending 31 May 1968, the north jetty was extended 800 ft into the ocean. A 452-ft-long timber weir, and a 280-ft-long rubble stone jetty (Figure 2) were also built. Construction of the southern stone jetty was pivoted to the north, thus offering protection



Figure 2. Rudee Inlet, VA, February 2002 (courtesy of Mr. Jay Bernas, city of Virginia Beach)

for dredging equipment operating in the inlet. The low weir section with top elevation at mean sea level (msl) was designed to allow the northward-directed longshore sediment transport to pass over it and into the sand trap (dredged to a depth of 20 ft below mean sea level (mlw)) between the weir and the inlet channel. The weir also acted as a wave break, giving protection to the dredging equipment in the basin (Needham and Johnson 1972). Tides at the inlet are semidiurnal with an average range of about 3.4 ft.

Participation in maintenance dredging by the Corps began in 1990. With the use of city dredge crews and a government dredge, clearing the ocean portion of the channel is performed up to four times a year. With this method, shoaling in the channel has been greatly reduced and the accumulation of sand updrift (to the south) of the sand trap has diminished (Beach and Waterways Advisory Commission 2002). Sand is bypassed downdrift to Virginia Beach, located north of the inlet. A December 2001 Rudee Inlet management study performed by the city of Virginia Beach found there is an average annual net northerly sand transport about 250,000 to 300,000 cu yd per year in the vicinity of Rudee Inlet.

Dredging of sand accumulation to the south of the inlet revealed deterioration of the timber weir and the inshore jetty anchoring the weir. The management study advised repairing infrastructure on the inlet's south side, sand-tightening of north jetty, and construction of north jetty spur. The Beach and Waterways Advisory Commission supports the findings of the study and the Environmental Assessment's Selected Plan became available in April 2002 to detail best management practices at the inlet.

WEIR JETTY SYSTEMS LOCATED IN SOUTH ATLANTIC DIVISION:

Masonboro Inlet, NC. This inlet has been open continuously since 1733. It migrated southward of its present position and, in 1909, it was located 4,000 ft downbeach from its present location. The protrusion of the south shoulder into the inlet in 1945 subsequently caused erosion of the north shoulder of the inlet. A channel was cut through the south spit, and three dikes were constructed at the south end of the separated spit in April 1947. Improvements for the inlet, authorized in 1949, included two jetties, an ocean entrance channel between the jetties, and interior navigation channel to the Atlantic Intracoastal Waterway (AIWW). The interior channels were dredged in 1957, and in 1959 the ocean navigation channel was dredged. The dredged material was placed across the southernmost channel so that flow would be concentrated through one channel with design depth of 14 ft at mlw and 400-ft width at the bottom. This channel shoaled quickly and was re-established in 1959 (Seabergh 1976).

Continued shoaling in the channel and attendant maintenance dredging problems led to the construction of the two previously authorized jetties. Lack of funds determined that the north jetty, due to its location on the apparent updrift side of the inlet, would be constructed first. Because of the predominant southerly littoral drift, the north jetty (Figure 3) was completed in 1965. In addition, the north jetty was designed with a weir to add a sand-bypassing feature to the overall navigation improvements. This was the first time that a sand weir bypassing feature had been incorporated into a USACE jetty design. The overall length of the jetty was 3,639 ft, consisting of 1,739 ft of concrete sheet pile and 1,900 ft of rubble mound on landward and seaward sections, respectively. The crest elevation of the shoreward 600 ft of the sheet pile varied from +12 to +2 ft mlw, with the 1,100-ft weir section at a crest elevation of +2 ft mlw, approximately at mean tide level. The rubble-mound

portion of the north jetty had design crest elevations of +6 ft mlw for 850 ft, a transition from +6 to +8 ft mlw for over 100 ft, and +8 ft mlw for the seaward 950 ft.

In 1969, because of the migration of the navigation channel toward the north jetty, a stone apron was placed to provide toe protection along the rubble-mound section of the jetty. This migration was not caused by the weir, but was a typical channel response to a single-jettied system (Kieslich 1981). Construction of the south jetty, built of quarystone and concrete sheet pile to a length of 3,450 ft, began in July 1978 and was completed in August 1980. In 1985, the south jetty was in good condition. The north jetty, however, was in need of repair work in several areas showing localized armor stone damage (Sargent 1988). Figure 3 shows the project. The bulbous right shoulder of the inlet formed after construction of the south jetty, when wave activity was reduced in that location. Previously, much sediment would enter Banks Channel (interior channel on right) and settle there to be dredged for beach renourishment of Wrightsville Beach. Thus the deposition basin region was not capturing sediment as designed. This sand eventually formed a spit and required vessels entering the inlet to make sharp turns in strong crosscurrents to remain in the navigation channel. However, by using the sand spit as an extended deposition basin area, the U.S. Army Engineer District, Wilmington, has only had to dredge the deposition area every 3 to 4 years, and the dredging also keeps the spit from further encroaching into the navigation channel (U.S. General Accounting Office (USGAO) 2002).

Little River Inlet, SC. This project was designed as a dual weir jetty. However, the rock weir sections were covered with armor stone to higher elevations, so the project is presently functioning as a normal dual jetty system (Figure 4). The plan was to observe shoreline response before removing the armor stone to create a single or dual weir jetty system.

Frequent shifting and migration of the barred channel and extensive sand shoals made the inlet dangerous for navigation. Construction of two rubble-mound jetties was started in March 1981 and completed in July 1983 at a cost of \$5.5 million. Total lengths of the upcoast and downcoast jetties were 5,660 ft and 8,815 ft, respectively (Sargent 1988). Each jetty consisted of a sand dike to anchor the structure to the shore, a sand-tight jetty section joining the weir to the sand dike, and a 650-ft weir section. The weir section is aligned at an angle approximately 45 deg with the updrift beach (Weggel 1981). Because the longshore sediment transport was considered variable, the weirs were covered with armor stone with the intent of removing the stones, if over a period of several years, excessive deposition of sand occurred. The jetty spacing at the parallel seaward ends was 1,000 ft. The minimum crest elevation of the structure was 8 ft mlw (exclusive of the weir section). The inlet entrance channel has a depth of 12 ft, is 3,200-ft long, and 300-ft wide across the ocean bar, and an inner channel 10 ft deep, 9,050 ft long, and 90 ft wide from the entrance channel to the AIWW.

Based on observations made in April 1974, the pre-project tidal prism was 505 million cu ft for the mean ocean tide range of 5.0 ft over a 12.42-hr tidal cycle. The crest elevation for the 1,300-ft-long weir section was +0.61 m (+2.4 ft) mlw at the mean tide. Analysis results indicated that relative balance in the fillet and shoal system did not warrant benefits from uncovering either of the weirs (Chasten 1992). An interesting result of the model study for this project was that use of weirs would permit shorter jetty structures, terminating in 8-ft depth instead of 10-ft depth (Seabergh and Lane 1977).



Figure 3. Masonboro Inlet, NC, 16 May 2002 (courtesy of Mr. William Dennis, U.S. Army Engineer District, Wilmington)

Murrell's Inlet, SC. Murrells Inlet had migrated up and down the coast over a range of 6,600 ft during the last century. To stabilize the inlet location, in 1977 two armor-stone jetties (Figure 5) were constructed at a cost of \$7.4 million. The north jetty, 3,455 ft long, consists: of the 560-ft-long shoreward jetty trunk; a 1,350-ft-long armor stone weir section (crest elevation +2.2 ft mhw, set at mean tide level) (Weggel 1981); the 1,650-ft-long seaward jetty trunk; and the 150-ft-long head section. The north jetty (with the exception of the weir section) and the south jetty are built to an elevation of 9 ft above mhw. The south jetty, 3,319 ft long, consists of a 3,170-ft-long trunk and a



Figure 4. Little River Inlet, SC, 1984

150-ft head section. Also, a 8-ft-wide fishing walkway was constructed on the crest of the jetty to elevation +10 ft mlw. Sand dikes composed of dredged material tied the jetty roots into the existing dune lines. The seaward parallel sections of the jetties were 600 ft apart with an entrance channel 300 ft wide and 12 ft deep at mlw, between them. The inner channel was dredged to a depth of 10 ft mlw and the deposition basin to a depth of 20 ft mlw with a capacity of 600,000 cu yd. It was dredged adjacent to the low weir section of the north jetty on the inlet side (Perry, Seabergh, and Lane 1978). More than a million cubic yards of sand from these two dredging projects were pumped to beaches in Garden City and Huntington Beach State Park.

The dominant direction of longshore transport was assumed to be southerly when the inlet project was designed (Douglass 1987), and the weir section was placed on the north jetty. As of 1985, the jetties had no history of damage or repair and appeared to be functioning properly (Sargent 1988).

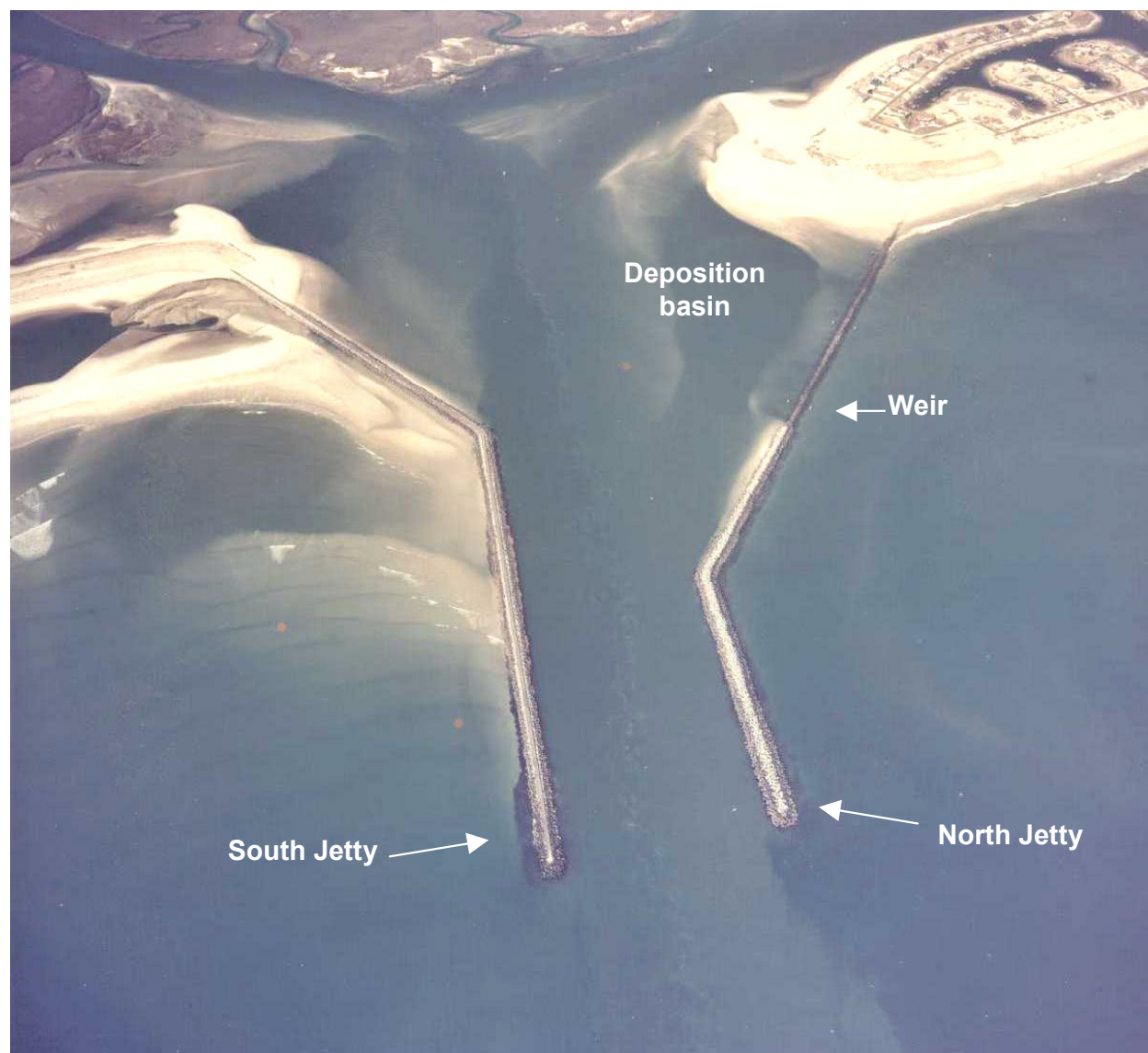


Figure 5. Murrells Inlet, SC, 14 January 1982

Spit growth into the bay along the inlet shoulder illustrates the usual path of sediment transport coming over the weir. Predominant waves and longshore current move sediment (fine sand) along the shoreline generating spit growth around the north shoulder of the inlet. Sediment is only entering the shoreward portion of the deposition basin. The ebb current also carries sediment from the edge of the spit entering the interior navigation channel into the bayward end of the deposition basin, as can be noted in Figure 5. According to the U.S. Army Engineer District, Charleston, the project has performed as expected because planned 3-year maintenance dredging has been needed only once since the project was built. District staff states the channel through the inlet has been kept open primarily by the flushing action of currents flowing through the jetties (USGAO 2002). The shoreline adjacent to the weir has receded an average of 2 ft/year since 1981.

Charleston Harbor, SC. Charleston Harbor is the site of a major commercial port and a U.S. Navy Base. Navigational problems such as inadequate water depths throughout the entire area, and maintenance dredging to keep the access channel at their prescribed depth because of shoaling, were experienced in the harbor (USAED, Charleston, 1979). Between 1878 and 1893, two converging jetties followed by a parallel section were constructed out from the barrier islands on each side of the harbor entrance. The nearshore portion of both jetties was constructed to -13 ft below the low tide water surface elevation, thereby serving as a weir and allowing the flood tide to enter. During the ebb tide, the bottom currents were channeled through the parallel section (constructed higher, with the seaward quarter above high tide) toward the bar, and this scouring action kept the new channel clear (Lockhart and Morang 2001). The total lengths of the north and south jetties were 15,400 ft and 19,100 ft, respectively. The distance between the parallel

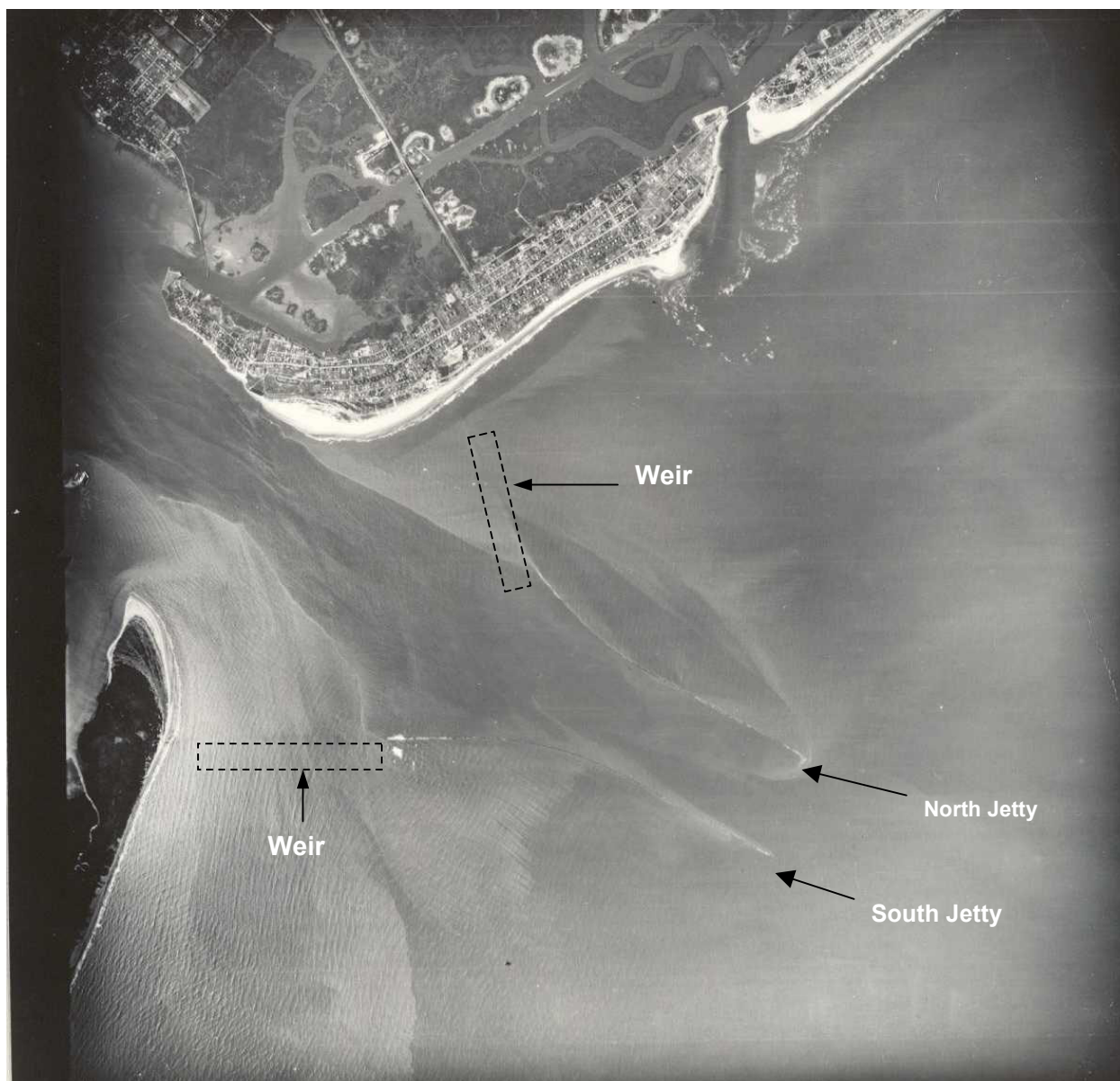


Figure 6. Charleston Harbor, SC, 1968

seaward sections of the jetties was 2,900 ft. Shoreward portions of both jetties, each approximately 6,000 ft long, were built up to typical elevations of -4 ft to -12 ft mlw. A typical section of the raised jetties consisted of a log and brush mattress foundation loaded with 30 to 60 tons of small stone weighing 10 to 250 lb. An additional narrow course of small stone was placed, and 1 to 7-ton granite blocks were placed as cover stone.

According to information found at the Charleston District Web site, the design of the existing channels in Charleston Harbor was inadequate to accommodate the great changes in Charleston's vessel fleet and rapid growth of commodity traffic that has occurred during the last decade. In 1991, the main channel was deepened to 40 ft. However, the volume of containerized cargo shipped through Charleston has increased, thereby causing container ships more than 200 ft longer than existing channel design to frequent the harbor. A feasibility study was conducted in 1996, and recommendations were authorized in the Water Resource Development Act. Deepening 16.3 miles of entrance channel to 47 ft deep and interior channels and turning basin to 45 ft are a portion of the authorized plan improvements listed in this study.

Ponce de Leon Inlet, FL. Prior to 1968, Ponce de Leon Inlet had functioned as a natural passage through the barrier islands separating the Atlantic Ocean from Halifax River and Indian River North. The inlet became recognized as difficult and dangerous to navigate with controlling depths over the ebb shoal of typically 4 to 6 ft (King et al. 1999). Dating to 1968-1972, two rubble-mound jetties



Figure 7. Ponce de Leon, FL, 13 December 1995

were constructed to provide safe passage via a 15-ft-deep by 200-ft-wide dredged channel. The north and south jetty have overall lengths of 4,050 ft and 4,078 ft respectively. A 1,800-ft-long weir of king piles and adjustable concrete beams was constructed along the north jetty. The weir section consisted of 300-ft length at +4 ft mlw, and 1,500-ft length at 0.0 mlw. An impoundment basin to entrap material passing over the weir in the north jetty was dredged between the weir section and the entrance channel (Taylor and Yanez 1994). In 1972, a year after the north jetty construction was completed, riprap was placed along the south side of the area adjacent to the weir section to provide scour protection.

In 1984, the entire weir was closed because the impoundment basin was ineffective, and navigation into the inlet was difficult due to the increased wave energy and crosscurrents in the inlet (Harkins, Puckette, and Dorrell 1997). The north jetty continued to experience scour, while the shoreline just south of and adjacent to the north jetty continued to erode westward. According to interpretations made by Vemulakonda et al. (1999), the channel, now restricted by jetties, is returning to pre-jetty orientation within the confines of the structures that force the channel to the north jetty. The flood-shoal channel has been dynamic, with large changes in flood-channel position and has dominated as the inlet has evolved. The mean tidal range offshore of Ponce de Leon Inlet is 4.1 ft and 2.3 ft at the U.S. Coast Guard station inside the inlet. Offshore mean spring tide range is 4.9 ft. Inside the inlet, mean spring tide range is 2.7 ft.

St. Lucie Inlet, FL. In 1892, St. Lucie Inlet, located at the south end of Hutchinson Island, was reported as being cut through the barrier island by local residents, 30 ft wide and 5 ft deep (Sargent 1988), but by 1922 it had widened to 2,600 ft (Marino and Mehta 1986). Mean-tide range is 2.6 ft. From 1926 to 1929, the north jetty was constructed out of coquina rock to a length of 3,325 ft. In 1966, Federal legislation was passed modifying the St. Lucie project to include maintenance of a 6-ft depth and 100-ft width channel between the Federal bar-cut channel and the AIWW.

During 1974-1980, the north jetty was extended 650 ft. A weir section was inserted to allow sediment to pass through it and accumulate in a spit behind the barrier island from where it could be safely dredged. In 1982, a 900-ft dogleg extension was added to the existing north jetty and partial excavation of an impoundment basin inside the inlet adjacent to the north jetty. A 450-ft-long detached breakwater was constructed near the center of the inlet to shelter the navigation channel. A south jetty, approximately 1,500 ft in length, was also constructed. In 1985, although structurally sound, the inlet was functionally unsatisfactory due to maintenance dredging for required channel depth (Sargent 1988). More sand accumulated in the navigation channel than was anticipated because the Corps did not complete construction of the deposition basin that was to collect the sand passing over the weir (USGAO 2002). In 1994, a sand-tight groin about 450 ft long at an elevation of about 4 ft National Geodetic Vertical Datum (NGVD) was constructed about 50 ft north of and parallel to the north jetty. Figure 8 shows the inlet in 1996. According to St. Lucie Inlet Federal Navigation Project, (Martin County, FL, 2002) continuation of Federal improvements were to occur during the summer of 2002 with the expansion and deepening of the impoundment basin to a 20-ft depth, raise outer 450-ft north jetty to 8.0 ft mlw, extend the south jetty by 200 ft, and sand tighten the north jetty landward of the weir section.



Figure 8. St. Lucie, FL, 23 November 1996

Boca Raton Inlet, FL. Boca Raton Inlet, located in southern Palm Beach County, FL, was a natural inlet subject to closure and reopening during storms because of sand shoaling. The city assumed maintenance dredging efforts in 1972 to keep the inlet navigable. In 1975, the north jetty of the inlet was extended 180 ft seaward (Marino and Mehta 1986). Although this modification improved navigability of the inlet by reducing shoaling, it accelerated beach erosion south of the inlet. In January of 1980, a weir section 65 ft long was constructed in the north jetty to allow a portion of the sand accreting on the north beach to be transported south. The south jetty was extended landward to prevent flanking (Palm Beach County, FL). The seaward edge of the weir was designed to abut the timber beam crib section of the north jetty, preserving the jetty extension intact; the 65-ft- wide weir intercepted the 1975 beach profile to the west and the north jetty crib to the east, at elevation 0 ft NGVD. The weir has achieved the desired balance between erosion-control on the beach south of the inlet while retaining navigable conditions in the inlet.

The inlet remains navigable at depths of 8 to 10 ft NGVD in the channel (Spadoni, Beumel, and Campbell 1983). Figure 9 shows the inlet system and Figure 10 shows a plan drawing. In 2001, the Bureau of Beaches and Coastal Systems adopted primary recommendations given in the Boca Raton Inlet Management Plan, 1992, to continue maintenance dredging of inlet, construct deposition basin, maintain existing jetty structures, and continue a beach/inlet monitoring program (Coastal Planning & Engineering, Inc. 1992).

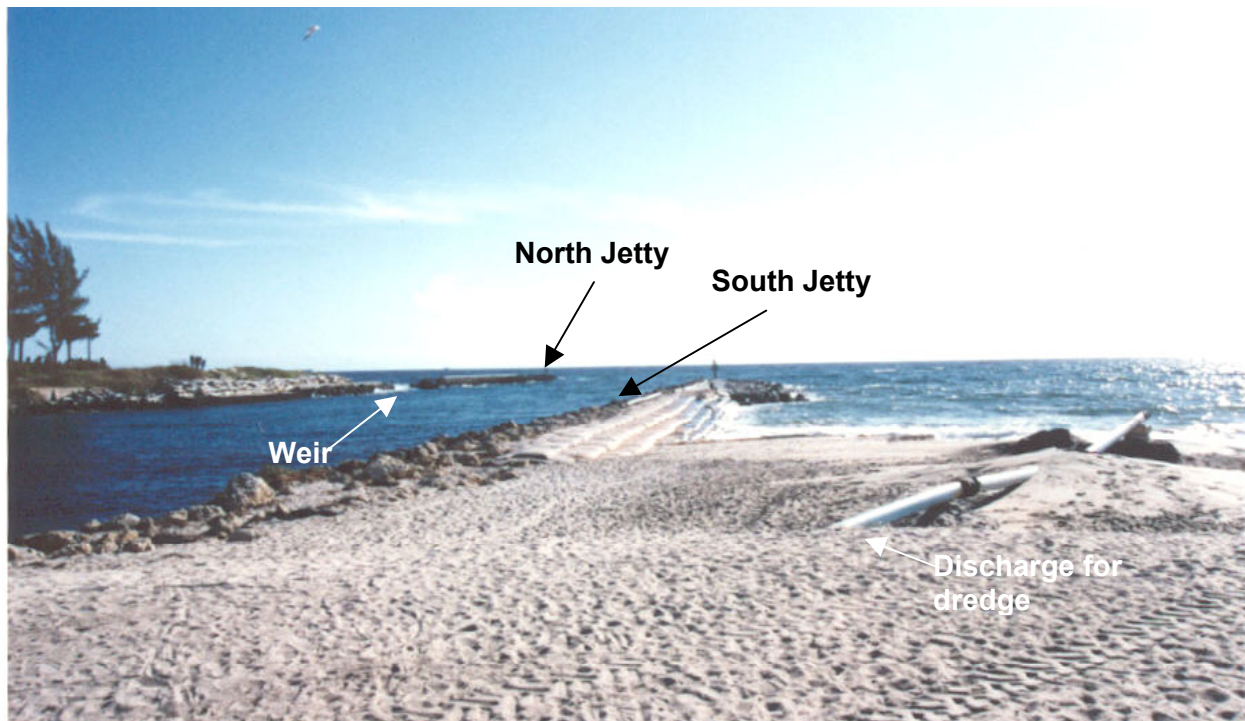


Figure 9. Boca Raton Inlet, FL, 1996

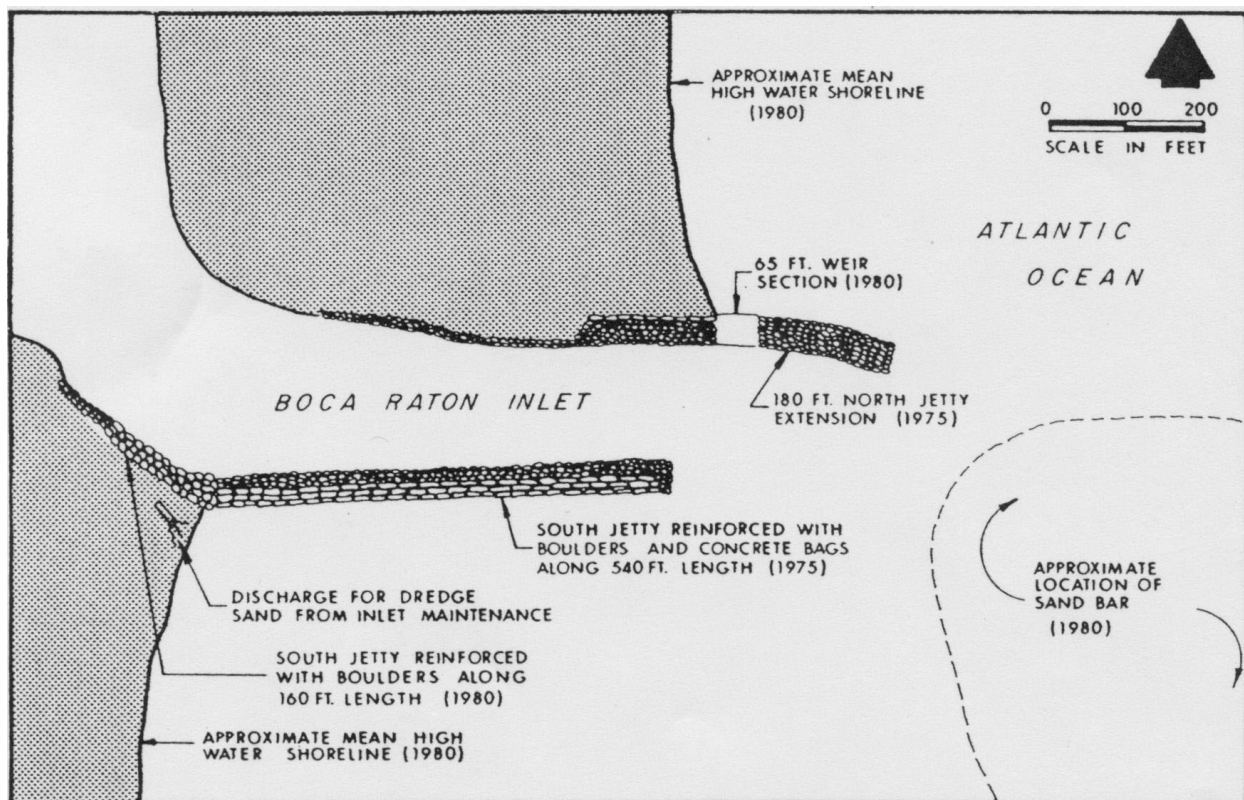


Figure 10. Weir jetty system, Boca Raton Inlet, FL

Hillsboro Inlet, FL. Hillsboro Inlet is a natural inlet about 2,000 ft long connecting to the AIWW. Shoaling was a problem for this inlet, thus causing navigational difficulties. In 1930, a granite rock jetty was constructed, projecting 260 ft southeast from the lighthouse to an existing rock reef formation. In 1952, a 500-ft-long timber jetty was built on the south side of the channel. Due to inadequacy in protecting the inlet from wave damage and also decaying timber, in 1964 a 35-ft cribbed rock structure was built to reinforce the south shore of the inlet channel. A 225-ft breakwater extension was built on the north side of the inlet in 1965. This formed a weir section, 260-ft-long (elevation +0.5 to +3.5 ft mlw) between the original jetty and the extension structure (Figures 11 and 12). The weir provided a means for sand to pass into the inlet deposition basin, thus making it the prototype for the weir-jetty concept (Weggel 1981). The mean tidal range is 2.3 ft and spring range is 2.7 ft (Brannen 1964). Predominant littoral drift is from north to south (University of Florida 1965). After completion of the jetty improvements, the inlet channel was deepened to improve navigation. This included a cut through the existing rock at the mouth of the inlet to a depth of approximately 10 ft below msl and width of 175 ft between the jetties. All inlet improvements were completed in November 1965.

These structures have been successful in halting the meandering of the inlet. The shore-parallel weir arrangement (Figure 12) proved beneficial due to limited wave action in the deposition basin area behind the weir because of the flat beach slope in front of the weir (Weggel 1983). They also have provided a means for regular dredging of the basin without exposing the dredge to harsh open-water conditions (Coastal Planning & Engineering, Inc. 1991).



Figure 11. Hillsboro Inlet, FL, 1 September 1995

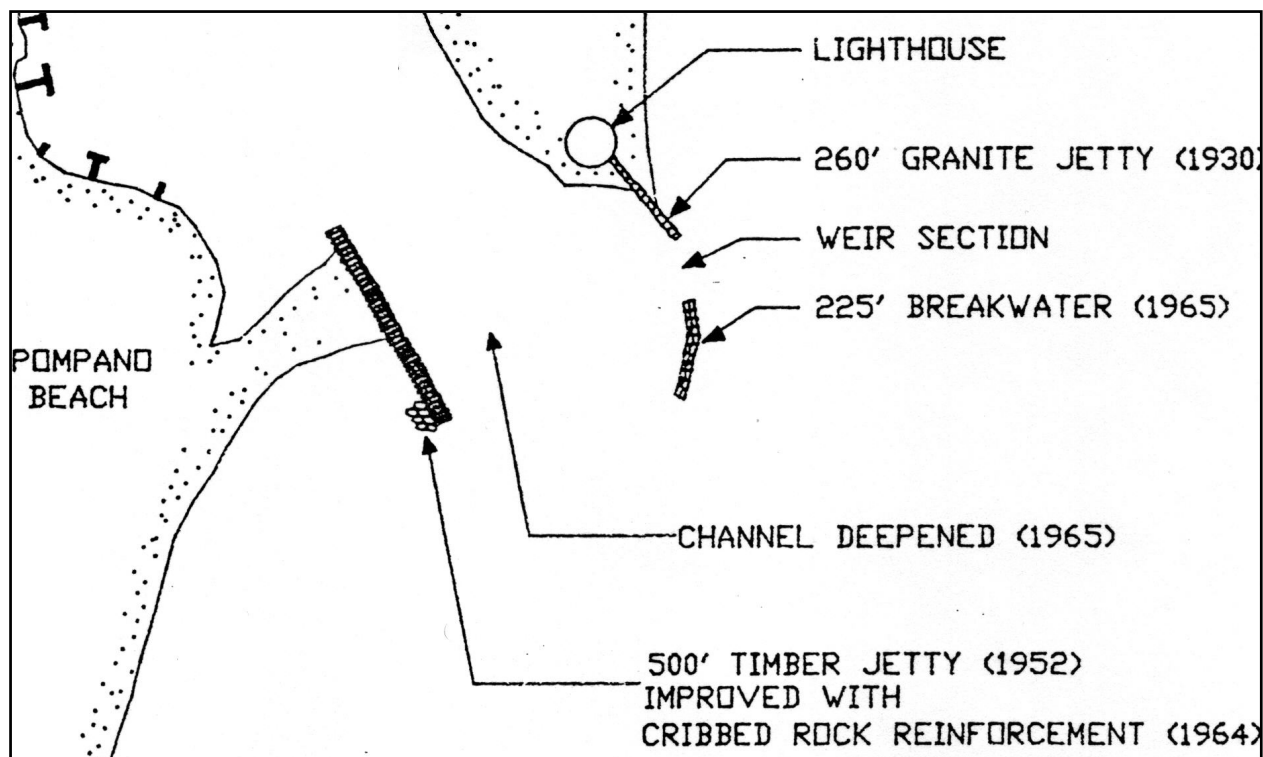


Figure 12. Hillsboro Inlet plan view

East Pass Channel, FL. In April 1923, the present East Pass Channel, connecting Choctowhatchee Bay with the Gulf of Mexico, opened as a result of a severe storm with high tides. Jetty construction and dredging began in December 1967 and ended in January 1969. Twin jetties were constructed, extending from each shore of the inlet to about the -6-ft mhw depth contour and spaced 304.8 m (1,000 ft) apart at their seaward ends. The 4,850-ft-long west jetty consisted of 1,200 ft of sand dike at the landward end, followed by 900 ft of rubble mound, followed by the 1,000-ft-long sheet pile weir, and ending with 1,750 ft of rubble mound. The weir was placed in the west jetty near the landward end to allow eastward-directed longshore transport to enter a deposition basin on the east side of the weir. The weir had -0.5-ft elevation and the diurnal tidal range was 0.6 ft (Weggel 1981). The deposition basin was dredged to provide storage for a 2-year supply of sediment, an estimated volume of 300,000 cu yd (Gosselin, Craig, and Taylor 1999). The 2,270-ft-long east jetty consisted of 1,270 ft of sand dike, followed by 1,000 ft of rubble mound.

Because westward-directed transport was entering the channel during flood tide and being deposited within the inlet, and also the eastward-directed transport appeared to be much smaller than expected, in 1977 the jetties were rehabilitated (Sargent 1988). The west jetty, seaward of the weir section had minor changes in cover stone, and the east jetty was modified with a 300-ft-long rubble mound spur that attached at a right angle to its landward end. The purpose of the groin was to divert the flow of the inlet away from the landward end of the east jetty because the beach directly to the north had been cut back (Morang 1992). In a Reconnaissance Report (U.S. Army Engineer District, Mobile, 1983), a recommendation was made to close the west jetty weir section (the net littoral drift was



Figure 13. East Pass Channel, FL, 1973

found to be from east to west with significant reversal), to reduce the shoaling in the inner channel areas caused by westward-directed transport passing through the weir section. Also, it was believed waves passing over the weir were causing erosion along the local shoreline. The weir was closed in 1985 by covering it with a rubble-mound trunk section identical to that placed on the rest of the jetty.

Perdido Pass, AL/FL. During 1968-1969, two converging jetties spaced 600 ft apart were constructed as part of a weir jetty system to help stabilize the natural inlet at Perdido Pass (Figure 14). The west jetty, 1,800 ft long, was of rubble-mound construction and extended from the south end of a vertical seawall constructed by the Alabama Highway Department. The east jetty, also 1,800 ft long, consisted of 1,290 ft of steel-reinforced concrete sheet pile and 560 ft of rubble mound (50 ft of overlap between the two sections). The east jetty has a lower weir section, 1,000 ft long with a top elevation of -0.5 ft mlw, that allows sand to pass over the rocks into a deposition basin in the pass when sand is moving westward in the wave-driven littoral current of Perdido Key (Douglass 2001). Diurnal tidal range was 0.6 ft (Weggel 1981).

During Hurricane Frederick, 12 September 1974, approximately 50 ft of material flanking the weir was lost, forming a channel between the weir and the beach. Dredged material was used to close the breach. In 1981, the jetty was rehabilitated, and in addition a rubble-mound section, 200 ft long, was added to the then existing landward end of the sheet-pile weir. The Mobile District stated this project was designed for dredging of about 100,000 cu yd of material annually; however, to keep the navigation channel at authorized depth, actual dredging was about 361,000 cu yd every 2 to 3 years (USGAO 2002).



Figure 14. Perdido Pass, AL/FL

WEIR JETTY SYSTEM LOCATED IN SOUTHWESTERN DIVISION :

Mouth of Colorado River, TX. The mouth of the Colorado River tended to skew to the west, most likely caused by response of the channel to spit growth from the east (Figure 15). As part of navigational improvements, dating 1988-1990 two rubble-mound jetties were constructed at the mouth of the Colorado River, providing protection for the 15-ft-deep, 200-ft-wide entrance channel, that discharges to the Gulf of Mexico. The west jetty is 1,450 ft long, and the east jetty, which contains a rubble-mound weir, is 2,650 ft long. The east jetty weir section begins 410 ft from the landward end and extends 1,000 ft seaward. The crown elevations of the weir section and remaining jetty sections are 0.0 ft and +8 ft mhw, respectively. The seaward ends of the east and west jetties are about 1,800 ft apart. The jetty design incorporated an impoundment basin on the channel side of the

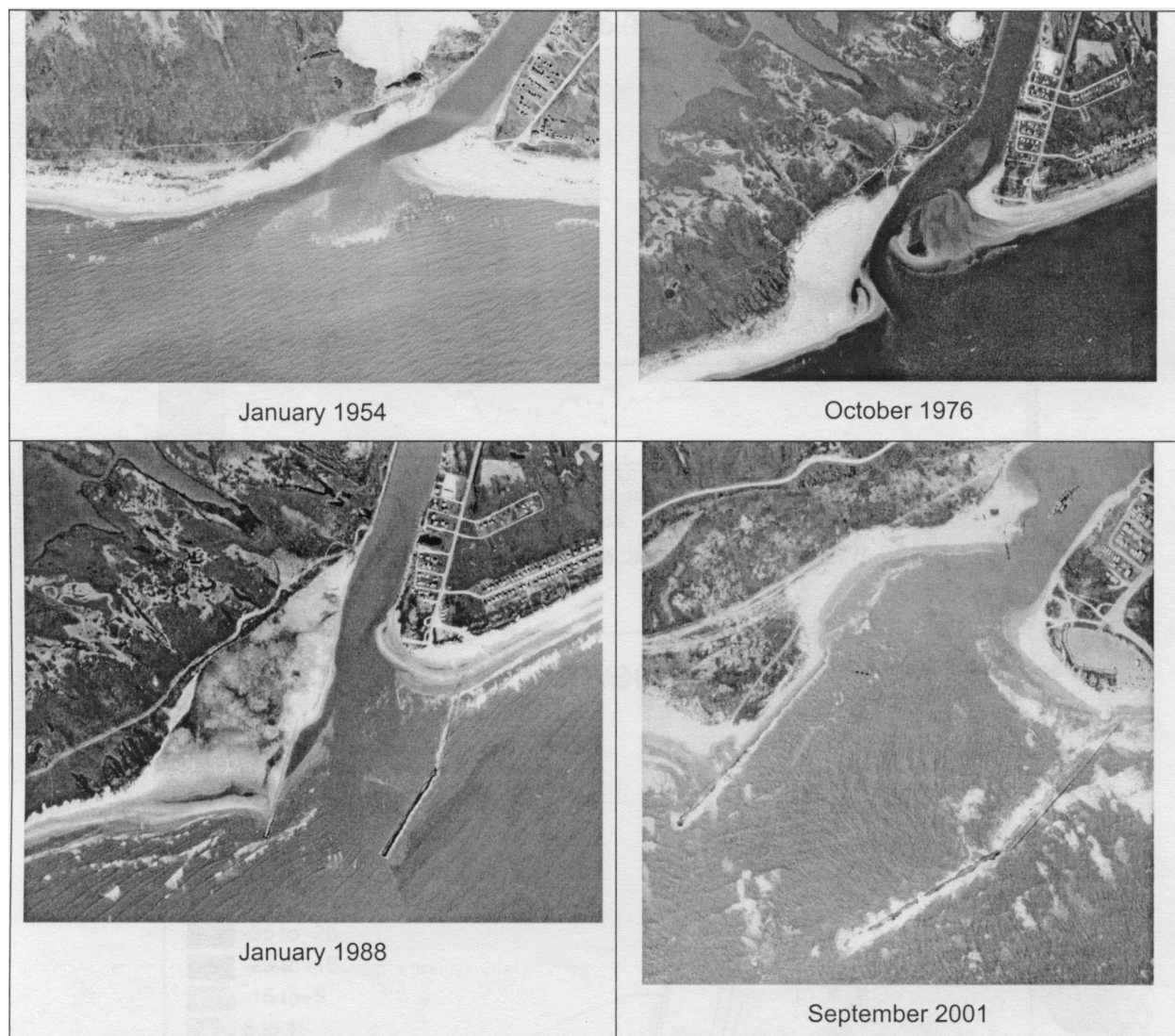


Figure 15. Colorado River Navigation Channel, TX

north jetty to trap littoral drift material passing over the weir (Sargent and Bottin 1989). The basin was initially dredged to hold a 2-year supply of 600,000 cu yd (Lin, Kraus, and Barack 2002).

After construction of the jetties and weir, the position of the channel became stable, and vessels could exit heading into the typical waves as opposed to meeting them broadside as would occur without the jetties. This weir jetty and impoundment basin were constructed to accommodate the anticipated increased deposition and have not functioned satisfactorily (Kraus, Lin, and Barack, in preparation), because the amount of sand flowing over the weir has been greater than expected, and it had been deposited in the navigation channel rather than the deposition basin (USGAO 2002). However, modifications to improve the functioning in the system are soon to be incorporated. The first modification is to construct an impoundment basin training structure to function as a groin in halting encroachment of the east spit to the channel and to direct westward moving longshore sediment transport into the basin (Kraus, Lin, and Barack, in preparation).

CONCLUSION: The weir jetty concept has been applied in a number of United States locations with various degrees of success. Factors contributing to performance below that intended include: sediment not being trapped in deposition basin (sediment pathway not intercepted by basin), waves passing over the weir, causing navigational difficulties inside inlet entrance, erosion of the interior beaches and increased sedimentation in the channel. Some projects have been successful and have aided in maintaining reliable navigation channel conditions by enhancing net seaward flushing of sediment in the channel, providing accessible locations for sediment bypassing, and minimizing the length of jetty required.

Table 1 summarizes information for weir jetty projects discussed in this CHETN. Presented are parameters describing the weir such as, elevation, length, composition, and orientation. Volume measurements of the deposition basin and gross longshore transport are also given along with the tidal prism and range.

Table 1 Weir Jetty Projects Specifications						
Location	Weir Elevation, ft mhw	Length of Weir, ft	Length of Weir Jetty, ft	Orientation of Weir, ¹ deg	Weir Material	Gross Longshore Transport Rate Estimates cu yd/year
Boca Raton, FL	0 ft (NGVD)	65	North jetty 650	90	Concrete bags	Net rate 120,000 to S
Charleston, SC	-13	6,000	15,400 N 19,100 S	70	Rubble mound	--
Colorado River Mouth, TX	0.0	1,000	East jetty 2,650	90	Rubble stone	300,000
East Pass Channel, FL	-0.5	1,000	West jetty 4,850	45	Concrete sheet pile	195,000
Hillsboro Inlet, FL	Varying +0.5 - +3.5	260	North jetty 485	0	Natural rock	120,000
Masonboro, NC	+2.0	1,100	North jetty 3,639	85	Concrete sheet pile	340,000
Murrells Inlet, SC	+2.2	1,350	North jetty 3,455	30	Rubble mound	250,000
Perdido Pass, AL/FL	-0.5	1,000	East jetty 1,800	45	Concrete sheet pile	195,000
Ponce de Leon, FL	0.0 +4.0	1,500 300	North jetty 4,050	60	King piles and adjustable concrete beams	700,000
Rudee Inlet, VA	+2.15	452	South jetty 815	90	Timber	600,000
St. Lucie, FL	0.0	900	North jetty 3,975	90	Rubble mound	260,000
Location	Tidal Prism, cu ft		Tidal Range, ft		Volume of Deposition Basin, cu yd	
Boca Raton, FL	--		2.8 spring		--	
Charleston, SC	5.75 x 10 ⁹		6.1 spring		--	
Colorado River Mouth, TX	1.75 x 10 ⁶		1.4 diurnal		740,000	
East Pass Channel, FL	1.62 x 10 ⁹		0.6 diurnal		150,000	
Hillsboro Inlet, FL	4.85 x 10 ⁷		Mean range 2.3 Spring range 2.7		--	
Masonboro, NC	6.8 x 10 ⁸		4.0 - 4.7		12,000,000	
Murrells Inlet, SC	5.8 x 10 ⁸		4.4 - 3.3		600,000	
Perdido Pass, AL/FL	4.356 x 10 ⁸		0.6 diurnal		--	
Ponce de Leon, FL	5.09 x 10 ⁸		2.7 spring		4,125,000	
Rudee Inlet, VA	1.66 x 10 ⁷		3.4 semi-diurnal		3,150,000	
St. Lucie, FL	5.15 x 10 ⁸		Mean range 2.6		15,750,000	
¹ Angle from general trend of shoreline measured on the channel side.						

ADDITIONAL INFORMATION: Questions about this technical note can be addressed to Mr. William C. Seabergh (601) 634-3788; e-mail: William.C.Seabergh@erdc.usace.army.mil. For information about the Coastal Inlets Research Program, contact the Program Manager, Dr. Nicholas C. Kraus at Nicholas.C.Kraus@erdc.usace.army.mil. This technical note should be cited as follows:

Seabergh, W. C., and Thomas, L. J. (2002). "Weir jetties at coastal inlets: Part 2, Case studies," ERDC/CHL CHETN-IV-54, U.S. Army Engineer Research and Development Center, Vicksburg, MS, <http://chl.wes.army.mil/library/publications/chetn>

REFERENCES

- Beach and Waterways Advisory Commission. (2002). "Beach Management Plan," City of Virginia Beach, VA.
- Brannen, T. E. (1964). "Hillsboro Inlet, Florida." *Meeting 50th of the Committee on Tidal Hydraulics*. Engineering Division, Jacksonville District.
- Chasten, M. A. (1992). "Coastal response to a dual jetty system at Little River Inlet, North and South Carolina," Miscellaneous Paper CERC-92-2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.
- Chasten, M. A., and Seabergh, W. C. (1992). "Engineering assessment of hydrodynamics and jetty scour at Little River Inlet North and SC," Miscellaneous Paper CERC-92-10, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.
- Coastal Planning & Engineering, Inc. (1992). "Boca Raton Inlet management plan," Boca Raton, FL.
- Coastal Planning & Engineering, Inc. (1991). "Hillsboro Inlet management plan, study phase report," Boca Raton, FL.
- Douglass, S. L. (1987). "Coastal response to navigation structures at Murrells Inlet, South Carolina," Technical Report CERC-87-2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.
- Douglass, S. L. (2001). "State of the beaches Alabama: 2001," Civil Engineering Department, University of South Alabama, Mobile, AL.
- Gosselin, M. S., Craig, K. R., and Taylor, R. B. (1999). "DMS:Diagnostic Modeling System, Report 2, Reduction of sediment shoaling of the entrance channel at East Pass, Florida," Technical Report CHL-99-19, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.
- Harkins, G. S., Puckette, P., and Dorrell, C. (1997). "Physical model studies of Ponce DeLeon Inlet, Florida," Technical Report CHL-97-23, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.
- Kieslich, J. M. (1981). "Tidal inlet response to jetty construction," GITI Report 19, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.
- King, D. B., Smith, J. M., Militello, A., Stauble, D. K., and Waller, T. N. (1999). "Ponce DeLeon Inlet, Florida, site," Technical Report CHL-99-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.
- Kraus, N. C., Lin, L., and Barack, R. G. (in preparation). "Alternatives for reducing shoaling at the mouth of the Colorado River Navigation Channel, Texas." *Proc. Dredging '02*, American Society of Civil Engineers.
- Lin, L., Kraus, N. C., and Barack, R. G. (2002). "Predicting hydrodynamics of a proposed multiple-inlet system, Colorado River, Texas." *Proc. Coastal and Estuarine Modeling Conference '02*. ASCE, 837-851.

Lockhart, J., and Morang, A. (2001). "Chapter 3: History of coastal engineering." In *The Lowcountry Engineers, Military Missions and Economic Development in the Charleston District*, by J. W. Moore, and D. P. Moore (1982), U.S. Army Corps of Engineers, 16-21.

Marino, J. N., and Mehta, A. J. (1986). "Sediment volumes around Florida's east coast tidal inlets," University of Florida, Gainesville, FL.

Martin County, FL. "St. Lucie Inlet Basin and Jetty improvements," Webserver.martin.fl.us/inlet. (23 Oct 02).

Morang, A. (1992). "A study of geologic and hydraulic processes at East Pass, Destin, FL," Technical Report CERC-92-5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Needham, B. H., and Johnson, R. E. (1972). "Historical review of Rudee Inlet," Technical Report No. 9, Institute of Oceanography, Old Dominion University, Norfolk, VA.

Palm Beach County, FL. "History of Palm Beach County Inlets,"
http://www.pbcgov.com/erm/divisions/enhancement/shoreline/pdf/history_inlets.pdf. (23 Oct 02).

Perry, F. C., Seabergh, W. C., and Lane, E. F. (1978), "Improvements for Murrells Inlet SC," Technical Report H-78-4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Sargent, F. E. (1988). "Case histories of Corps breakwater and jetty structures: Report 2, South Atlantic Division," Technical Report REMR-CO-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Sargent, F. E., and Bottin, R. R. (1989). "Case histories of Corps breakwater and jetty structures: Report 9, Southwestern Division," Technical Report REMR-CO-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Seabergh, W. C. (1976). "Improvements for Masonboro Inlet, North Carolina," Technical Report HL-76-4, Hydraulics Laboratory U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Seabergh, W. C. (1983). "Weir jetty performance, hydraulics and sedimentary considerations," Technical Report HL-83-5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Seabergh, W. C. (2002). "Weir jetties at coastal inlets: Part 1, Functional design considerations," ERDC/CHL CHETN-IV-__, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Seabergh, W. C., and Lane, E. F. (1977). "Improvements for Little River Inlet South Carolina," Technical Report HL-77-21, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Spadoni, R. H., Beumel, N. H., and Campbell, T. J. (1983). "The design and performance of a weir at Baca Raton Inlet, Florida," *Coastal Structures* 894-901.

Taylor, R. B., and Yanez, M. A. (1994). "Ponce DeLeon Inlet management plan," Taylor Engineering, Inc., Jacksonville, FL.

United States General Accounting Office. (2002). "Oregon Inlet Jetty Project," Washington, DC

University of Florida (1965). "Coastal engineering study of Hillsboro Inlet, Broward County, Florida," Florida Engineering and Industrial Experiment Station, Coastal Engineering Laboratory, Gainesville, FL.

U.S. Army Engineer District, Charleston. (1979) "Benthic and sedimentologic studies on the Charleston Harbor ocean disposal area, Charleston Harbor deepening project," Marine Resources Division, Charleston, SC.

U.S. Army Engineer District, Mobile. (1983). "Reconnaissance Report, East Pass Channel, Destin Florida," Mobile, AL.

Vemulakonda, S. R., Scheffner, N. W., Mark, D. J., and Brown, M. E. (1999). "Water-surface elevation frequency estimates for the Louisiana coast," The CERCular, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Weggel, J. R. (1981). "Weir sand-bypassing systems," SR-8, U.S. Army Corps of Engineers, Coastal Engineering Research Center, Fort Belvoir, VA.

Weggel, J. R. (1983). "The design of weir sand by-passing systems," *Coastal Structures* 83 ASCE, 860-874.

NOTE: The contents of this technical note are not to be used for advertising, publication, or promotional purposes. Citation of trade names does not constitute an official endorsement or approval of the use of such products